VOLUME 79 SEPARATE No. 306

PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

OCTOBER, 1953

AN ENGINEERING APPROACH TO BLAST RESISTANT DESIGN

by N. M. Newmark, M. ASCE

Presented at
New York City Convention
October 19-22, 1953

STRUCTURAL DIVISION

(Discussion open until February 1, 1954)

Copyright 1953 by the AMERICAN SOCIETY OF CIVIL ENGINEERS
Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

THIS PAPER

--represents an effort of the Society to deliver technical data direct from the author to the reader with the greatest possible speed. To this end, it has had none of the usual editing required in more formal publication procedures.

Readers are invited to submit discussion applying to current papers. For this paper the final closing dead line appears on the front cover.

Those who are planning papers or discussions for "Proceedings" will expedite Division and Committee action measurably by first studying "Publication Procedure for Technical Papers" (Proceedings — Separate No. 290). For free copies of this Separate—describing style, content, and format—address the Manager, Technical Publications, ASCE.

Reprints from this publication may be made on condition that the full title of paper, name of author, page reference, and date of publication by the Society are given.

The Society is not responsible for any statement made or opinion expressed in its publications.

This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N. Y.

AN ENGINEERING APPROACH TO BLAST RESISTANT DESIGN

N. M. Newmark*, Member, ASCE

Summary

The field of structural dynamics is relatively new. It is therefore understandable why practically all of the work that has been done to date on the subject of the resistance of structures to atomic blast, has been concerned primarily with the analysis of the response (i.e. deflection) of a particular structure subjected to a given loading. This approach is mathematically very attractive, and the writer has contributed a number of methods of his own. However there are two important reasons why the calculation of deflection of a given structure, subjected to a given blast, is not very satisfying from an engineering point of view:

(1) The structure to be designed does not yet exist, therefore needless complication in the analysis merely lengthens the time it takes to select a structure for analytical trial. The real problem is the preliminary choice of the structure to be analyzed. If this can be done intelligently, the further analysis may actually be unnecessary.

(2) Even minor variations and uncertainties in the details of the blast loading or in the properties of the structure (such as yield point, stiffness, etc.) cause major changes in the computed structural response. This fact is not immediately evident from the usual detailed descriptions of analytical techniques, but it can readily be demonstrated. Consequently, the customary type of analysis may be grossly misleading. The designer is concerned not with what does happen in specific circumstances but with what might happen under a range of circumstances, and with the probability of the occurrence of the phenomena in question.

For these reasons the writer presents without apology the relatively crude approximations in this paper as a means of arriving at a design reasonably quickly and simply. For a preliminary design of either a structural frame, or component parts such as wall covering panels, one needs to make an estimate of the natural period of vibration of the component relative to the duration of the loading on the component, and the ductility factor or the ratio of the desired limiting deflection (preferably for "collapse") to the yield point deflection. Extremely accurate values are not required. From these estimates, one determines readily from the approximate relations herein the required "limit load " or yield resistance of the structure for a given external pressure or size of bomb, etc.

In other words, the procedure develops essentially an equivalent static loading (or resistance) for which the design must be made. Since this is a concept familiar to structural engineers in general, the procedure can be followed in almost any design office.

*Research Professor of Structural Engineering, University of Illinois

The inexperienced designer may have to make a revision in his estimate of the basic parameters after the preliminary design is complete. This can easily be accomplished, however. Finally, after the design is completed a calculation may be made by numerical procedures or other methods if more apparent precision is desired in the estimate of deflection corresponding to the selected loading, or for other loading conditions.

Basic Considerations

The designer must choose first the conditions governing the design. He may wish to make the structure adequate to resist the blast forces from the detonation of a given size of bomb at a given radial distance and a given height, or he may select several alternatives and insist that the structure be adequate for any of them. The designer may also select, merely as a standard criterion, a given shape of blast pressure-time curve, with specified values of peak pressure and duration of the positive phase of the pressures.

The relations among these various blast parameters are described in detail in Reference 1 and in papers by Lampson (Ref. 2); and by Reines, Bleakney, and Merritt (Ref. 3).

There is no real advantage to be gained in designing for a particular size of bomb, which implies a particular shape of pressure-time curve. There is, however, a good deal of advantage in considering a conventionalized loading so as to insure a uniform factor of safety against collapse in all the parts of a given structure, or in all the structures in a region. This is the standard approach used in design for earthquake resistance (Ref. 4) or to resist wind loads.

The type of overpressure-time curve developed in a blast is shown in Fig. 1 by the solid curve labeled p_s. This can conveniently be replaced by a corresponding linear curve varying from a maximum pressure p_o to zero in a time t_s slightly less than t_o. For most structures it is convenient and not inaccurate to neglect the negative phase of the blast beyond t_o.

The next choice to be made is that of the type of structure, whether it is to be completely enclosed with a covering which resists the blast forces and thereby shields the interior of the building, or whether it is to be essentially open, possibly covered with light, frangible siding that permits the blast to enter and pass through the structure. The forces acting on the structure and on its component parts differ greatly for these extreme types.

Of course, the structure may be intermediate in type with resistant covering having openings or windows which permit the blast to enter in part.

Finally, the designer must make a decision as to what point in the range of the behavior of the structure he wishes to consider as the maximum permissible response for the other selected conditions. He may choose to have the structure remain elastic (but only rarely because of the extremely heavy construction required), or he may choose to permit the structure to deflect to the point of imminent collapse, or he may select some intermediate range. In most cases, design for collapse is reasonable, since a factor of safety may be already implied in the choice of the pressure curve for which the design is to be made. This choice has to be made independently for the different parts of the structure, as the conditions are different for wall panels, structural framing members, and shear walls, for example.

This discussion dodges the question as to what constitutes "collapse"; this is a topic which requires a good deal of discussion. However, the writer would consider this as the point of complete failure of the structure or component, in general, in order not to overload the procedure with compounded factors of safety in too many hidden places.

When these various choices are made, the procedure leads to the computation of the yield-point resistance required (or possibly the peak overpressure required for the given yield resistance)). The designer must recognize that factors beyond his control governing the blast pressure, duration, structural properties, etc., may make his computed value uncertain by as much as 25 percent. However, nothing that he can do, particularly in refining his calculations, can reduce this uncertainty. It is inherent in the problem. Only complete and accurate physical tests on the completed structure, and complete and accurate pressure measurements during the loading of the structure by blast, can permit a more accurate calculation to be made.

This means that a structure designed barely to resist collapse may suffer only slight plastic deformation, or conversely, in the extreme case of variation. Account can be taken of this phenomenon in selecting the actual design in order to insure safety if an unusual degree of assurance is necessary.

It is clear from the foregoing reasoning that the uncertainty in the deflection, reached by a given combination of loading and structural resistance, is very much greater than the uncertainty in the pressure level (or in the required resistance). For this reason, throughout this paper, attention is focussed on the load or resistance required to reach a desired deflection which may or may not be achieved or exceeded.

Blast Loading on Structures

Reference has already been made to the free stream or side-on overpressure curve, in Fig. 1, which corresponds to the condition when no structure or other object interferes with or causes diffraction of the shock wave in the blast. Accompanying this and dependent on it, there is a particle motion of the air which exerts a dynamic pressure or "drag" force on objects which interfere with the flow of air. It is convenient in the following discussion to consider the drag pressure as corresponding to a drag coefficient of unity. It is also convenient although perhaps not entirely accurate to consider that the drag pressure at any instant has the value it would have if the overpressure p_s at that time were a steady-state value.

With these assumptions the value of p_d can be approximated by the relation:

$$p_d = 0.024p_s^2/(1 + 0.01p_s) \cong 0.022p_s^2 \qquad (1)$$

Equation (1) is adapted with only minor modification from Reference 5. The solid curve marked p_d in Fig. 1 shows the time variation of drag pressure corresponding to the values of p_s. In general, however, the drag pressure-time variation can be approximated by a straight line from the initial peak value to zero at a time t_d which is $0.5t_s$.

Diffraction-Type Structures.

When the shock wave from the blast encounters an obstacle such as the rectangular block in Fig. 2, the pressures on the block are related to, but different from, the pressures in the curves of Fig. 1. Immediately upon striking the front wall the pressure in the shock is increased by reflection, and the net reflected overpressure p_r is given by the relation:

$$p_r/p_s = 2 + p_s/(17.2 + p_s/6) \qquad (2)$$

or without serious error, for p_s less than 40 psi,

$$p_r = p_s(2 + p_s/20) \qquad (3)$$

As the shock traverses the building, the pressures change. The high pressure air trapped in front of the face of the building spills around and over the building permitting the pressure to drop. The time for the drop to occur is dependent on the smaller of the two quantities H or B/2, designated by S, the least distance from the "stagnation" point on the front face of the structure, to an edge. In Reference 5, several conventionalized relations are given for the net front face pressure. The curve shown extending from p_r in Fig. 2 is a representation of the results with slight amendments as indicated by the studies of Bleakney and Merritt in Reference 3. This curve indicates the pressure as dropping to a value which is the sum of the side-on pressure plus a drag coefficient for the front face (c_f) multiplied by the drag pressure, in a time 3S/U where U is the velocity of propagation of a disturbance in the shocked air, but which is taken herein for convenience as the shock front velocity also, and may be taken without introducing serious error in the results as about 1500 ft. per sec. More accurate values can be determined from standard text books or from Reference 1. The true value may range from 1130 ft. per sec. to upwards of 2000 ft. per sec. for strong shocks. However the undertainty in the other quantities appearing in the relations in which U appears makes it really unimportant to determine U precisely.

On the rear face the shock reaches the face at a time L/U later than that for the front face, and the average pressure on the rear face, directed rearward, changes in an increment of time of approximately 5S/U to a value corresponding to the side-on pressure less the drag coefficient for the rear face (c_r) multiplied by the corresponding drag pressure. These quantities are shown in Fig. 2 with a prime to indicate that they are measured from a different origin in time L/U later than the curves for the front face.

The difference between the forward pressure on the front face and the rearward pressure on the rear face is shown by the shaded area in Fig. 2, and by the solid curve in Fig. 3, up to the time t'_1 when the net translational force on the building reaches zero. However, if L is small, the pressure on the rear face never gets larger than that on the front face, and the whole building experiences a forward drag. Since p'_d is very nearly the same as p_d if L/U is small compared with t_d, then the drag coefficient C_d is the sum of c_r and c_f. The dash-lined curve in Fig. 3 indicates the appearance of the net translational force in this case. In general it is suggested that values of c_f and c_r be taken as 0.8, and C_d as 1.6, unless experimental values are available.

Except for the tail end of the curve where long-continued drag exists, the diffraction around the building leads to pressures which can be approximated reasonably well by a single triangle, as shown by the dot-dash line in Fig. 3, extending from p_r to zero in a time t_1, where t_1 is given by the approximate relation:

$$t_1 = (L + 4S)/U - L\,(L + 10S)/\,2U^2 t_s \tag{4}$$

Equation (4) is valid only if Ut_s is greater than 2L + 10S. A better value of t_1 can be obtained by plotting the curve of net loading as in Fig. 3, but the value given in equation (4) is generally adequate.

For the pressure on a wall, the curve for the front wall of the building, in Fig. 2, can be used. If the sloping line is extended down to the base line, a value of t_1 can be obtained. This will be in general dependent on the pressure level. No more refinement is usually necessary because of the fact that maximum deflection in the wall panels of a building will usually occur very quickly, and consequently, the duration of the pressure is long enough so that

changes in the duration are not significant. An approximate relation for the corresponding value of $t_1 = t_f$ for a front wall panel is the following:

$$t_f = 6S/ (U + 3S/ t_s) \tag{5}$$

Since any wall may be exposed directly to a blast, the design conditions for all wall panels should be taken as the same. Also, the roof slab may be exposed to a force from a detonation above the roof, so that it too is exposed to reflected pressures. However, if it is clear that the roof is never exposed to such a condition, a value of p_o can be used instead of p_r for the roof pressure.

Some data on the pressure to be expected on the roof for such conditions can be obtained from the report by Bleakney in Reference 3. As a very crude approximation, however, one can take the pressures as given approximately by the maximum value of p_o, with a duration as indicated by equation (5).

Drag-Type Buildings.

For an open structure with only beams, columns, trusses, or other members with only small area opposing the blast, each of the members receives an impulsive loading as the blast engulfs it, and each member is then exposed to drag from the wind accompanying the blast. Because the blast is transmitted through the building in a finite time, the net translational force increases in general until all or nearly all of the building is engulfed. However, the impulse from the diffraction around each member produces a spike on the net force diagram as the blast reaches that particular member. Unless the building is extremely brittle, and fails without plastic deformation, it is reasonably accurate to consider that the building is subjected only to drag, neglecting the impulse spikes, but to compensate for this, consider that the whole building is engulfed at once.

This assumption leads to a net pressure on the projected area of each of the elements corresponding to the drag pressure multiplied by the drag coefficient for the individual members. The basic recommended pressure curves for use in design are shown in Fig. 4 for both diffraction-type and drag-type buildings.

The drag coefficients to be used should take into account the shielding of elements by others placed a short distance away. However, if the distance between parallel elements is more than 10 times their width, the shielding is probably negligible. Reference 5 indicates drag coefficients of about 2 for structural shapes, about 1.25 for box-shaped elements or for flat plates, and about 0.8 for cylinders. In the absence of better information these values can be used.

In Fig. 4, the value of p_{do} is obtained from Equation (1) when $p_s = p_o$.

Partly Open Buildings.

The question of buildings with resistant walls but with openings in the walls is a difficult one. Any comprehensive treatment would be beyond the scope of this paper. However, the following approximate treatment is suggested.

When the area of the openings is more than about 50 per cent of the area of the walls consider the building as a drag type building. This approach was suggested in Reference 5. In this reference, it was also suggested that where the area of the openings is less than 5 percent of the area of the walls, the building can be considered as completely closed.

For intermediate conditions, the magnitude of the pressure that gets through the opening must be considered. It is suggested, however, that in lieu of a better approach, a direct interpolation be made for conditions where

the openings are more than 5 percent and less than 50 percent of the wall area, the interpolation to be made both for maximum pressure and for duration.

Structural Resistance

The relation between load and deformation, or between resistance and deflection, of a structure or structural element may take any of a number of forms. It is convenient to define the structural resistance r in the same terms and in the same units as the external loading p. Then the relations between r and x may be indicated as in Fig. 5, where x is either a particular deflection, an average deflection, or a parameter which defines all of the deflections in terms of some mode shape.

In Fig. 5(a) there are shown a brittle and a ductile resistance curve. The former is defined by the fact that failure occurs in or near the linear range of the load-deflection relation.

When heavy dead loads or other large vertical forces in a structural frame deflect laterally they produce moments and stresses in the frame. These reduce the ability of the frame to resist lateral load. The effect is proportional to the deflection, and may be approximated on the resistance-deflection curve as shown by the dotted line in Fig. 5(a). The equivalent or effective resistance-deflection curve is the difference between the original curve and the dotted line. Consequently, the effective curve may have a net downward slope after yielding occurs and may even drop to zero resistance for a large enough deflection, at which point the structure would collapse physically of its own weight.

In Fig. 5(b) there are shown conventionalized curves which represent the behavior of a variety of structures. The figure shows (1) a work-hardening relation in which the resistance increases with increasing deflection beyond the first yielding, (2) an elasto-plastic relation with an ideal plastic constant resistance after yielding, and (3) two unstable relations with decreasing resistance after yielding. These may arise because of the effect of vertical loading.

In Fig. 5(c) a work hardening curve and an unstable curve are both approximated by equivalent elasto-plastic curves to give the same maximum deflection. It is shown later that the equivalent curves can be stated in fairly convenient form, and therefore it is only necessary to consider in detail the behavior of idealized elasto-plastic structures in order to obtain a rational analysis of practically any structure.

Finally, in Fig. 5(d) there is shown a typical resistance-deflection relation for an element that yields in successive stages, as for example a fixed-end beam. In such a structure the initial elastic relations apply up to the point where plastic behavior begins at the fixed end. Then beyond this point, the structure behaves for <u>additional</u> load as if it were hinged at the end, and the slope of the load-deflection curve is correspondingly greater than in the initial state. Finally, the beam yields also at the center and cannot carry a greater load. The limit resistance q is clearly defined; it is the maximum resisting capacity of the structure. However, it is less convenient in general to work with polygonal relations such as the one shown. Therefore an equivalent elasto-plastic relation is used, as shown by the dashed curve, with the proper yield point q, but with a fictitious yield deflection x_e which is chosen so that the area under the equivalent curve is exactly the same as that under the original curve, so that the energy stored is the same in both cases.

Required Resistance for Elasto-Plastic Structures

We have standardized or conventionalized our problem to the point where we have a structure with an equivalent elasto-plastic resistance-deflection relation loaded with a pulse loading having a maximum value p_{max} at time $t = 0$ and a duration t_1. (Or t_d, etc.) The yield resistance q is stated in the same terms as p_{max}.

The deflection has not been completely specified, nor do we need to specify it. We must only be sure that the maximum desired deflection or the collapse deflection x_m is measured in the same units and at the same point or points as the yield deflection at the knee of the curve, x_e. The ratio of the maximum to the yield deflection is defined as the ductility factor μ. For a completely brittle structure $\mu = 1.0$, for moderately brittle structures $\mu = 3$ to 5, for moderately ductile structures $\mu = 10$ to 30, and for very ductile structures μ is greater than 40.

Now we need only one other quantity to specify the essential characteristics of the structure or the component element. This is the natural period of vibration T. However, we must use the period corresponding to the stiffness of the equivalent elasto-plastic structure, as defined for example by the dashed line in Fig. 5(d) rather than as defined by the initial slope of the resistance curve.

The effective period is readily obtained from the period in the initial elastic stage by multiplying the latter by the square root of the ratio of the slope of the equivalent elastic resistance to that of the initial elastic resistance.

Standard procedures are available for the calculation of the period of the structure in its initial elastic state, either analytically or by numerical means. Again, only an approximate value is required.

With the simplifications outlined herein, the structure is essentially reduced to an equivalent simple one-degree-of-freedom system. Solutions of such systems have been obtained for a number of types of loading and for various types of resistance curves. Some of the results are given in Reference 6 in the form of charts. A simplified chart based on Reference 6 is given as Fig. 6. This chart relates the ductility factor or the ratio of maximum to yield deflection, the relative duration of the loading to the natural period of the structure, the ratio of the structural resistance to the peak loading, and the ratio of the time to reach maximum deflection to the natural period. The first of these quantities is plotted as horizontal lines with a logarithmic spacing, the second as vertical lines also with a logarithmic spacing, the third as lines sloping generally up to the right, and the fourth as dashed lines sloping generally down to the right. If any two of the quantities are given, the other two can be obtained.

For example, if $t_1/T = 2.0$, and $\mu = 20$, then it is readily found that:

$q/p_{max} = 0.6$ and $t_m/T = 1.7$.

Approximate equations for the items of interest are given later in this paper.

The relations governing the motion of a simple system can best be explored on a plot of the accelerations of the system as a function of time, as in Fig. 7. The accelerations produced by the external loading p are plotted as the triangle on the left hand side of the figure. The negative accelerations produced by the resistance r then are shown as the curve rising to a maximum and remaining constant after a time t_e. The part of the curve that is not known is the initial part of the resistance-time curve. This can be obtained by an iterative process, or it can be approximated quite readily as described in the paper by Newmark in Reference 2.

Now the value of t_m is obtained by the condition that the velocity at this time is zero. As a consequence the net area between the curves must balance, or

$$A_1 + A_2 = A_3$$

This means also that the area under the pressure curve and that under the resistance curve, between the origin and the time of maximum displacement, must balance. Further, the moment of the couple formed by these equal areas is equal to the maximum displacement.

When t_1 is very small, it doesn't matter what the shape is of the load-time curve; so long as its impulse is constant the maximum deflection and the time to reach it will be the same.

In order to take account of the speed of loading of the material, it is necessary to formulate an expression for the time to reach yielding. This can be done by making use of the observation that the yield displacement x_e occuring at time t_e is the moment about t_e of the net areas to the left of t_e. However, the moment of the resisting forces is small and can be neglected. Then, let $\bar{p}$ be the effective value of the pressure during the time up to yielding, and compute x_e as follows:

$$x_e = p\, t_e^{\,2}/2m$$

from which,

$$kx_e = q = \bar{p}t_e^{\,2}k/2m = 2\pi^2\bar{p}t_e^{\,2}/T^2$$

whence

$$t_e/T = 0.226\ (q/\bar{p})^{0.5} \tag{6}$$

This equation is valid if the duration of the pressure t_1 is greater than t_e, which is always the case for atomic bombs. If we designate the pressure at time t_e as p_e, then $\bar{p}$ is given by the equation:

$$\bar{p} = (2p_{max} + p_e)/3 \tag{7}$$

For a complicated structure we have the problem of defining the resistance q. Consider for example the structure in Fig. 8 consisting of several masses M each carrying an external force P and a resisting force Q applied by the structure. If we consider Q as being uniformly distributed, in the same way as the initial value of P or p, the deflection pattern will change in shape as Q increases. We can, however, take various values of Q and determine the deflections. We can then compute the stored energy as the summation of the quantities

over all of the masses in the structure.

We can then take the maximum deflection as a measure of the deformation and with that and the parameter defining Q obtain the quantities needed for the analysis. Alternatively we may take a pattern of displacement varied by multiplication by a parameter, find the corresponding individual values of Q and the stored energy, and by means of the plot at the bottom of Fig. 8 we can then define an effective or equivalent value of Q. The shape of the displacement pattern used will also serve to define the effective value of P in the same way as Q.

Empirical Relations

If now we consider the work done in general by the pressure and by the resistance, we can see that these must be equal if the mass of the structure is initially at rest, because the structure also stops temporarily at its position of maximum displacement. Also, if the pressure is applied very quickly as an impulse, the work done by the pressure is equal to the kinetic energy of motion of the structure after the external forces have ceased and before the internal resistances have been built up.

With these concepts we can derive the following results:

If the pressure is of infinitely long duration, the work done is equal to the internal energy stored at maximum deflection. Then we have

$$p_{max}x_m = q(x_m - 0.5x_e)$$

or

$$p_{max}/q = 1 - 0.5/\mu \tag{8a}$$

On the other hand, if the pressure lasts a very short time, the positive impulse of the pressure is $0.5p_{max}\, t_1$, and the initial kinetic energy of the mass is

$$0.25\, p_{max}^2\, t_1^2\, /2m$$

However, this is equal to the stored energy at maximum deflection, as before. Hence, with use of the relation

$$m = (m/k)\,(kx_e/x_e) = q\, T^2 / 4\pi^2 x_e$$

one can derive the result

$$p_{max}/\, q = (2\mu - 1)^{0.5}\, T/\pi t_1 \tag{8b}$$

Now, equation (8b) applies when t_1 is very small and equation (8a) when t_1 is very large. Trials with various combinations of the equations led to the following generally applicable result:

$$p_{max}/\, q = (2\mu - 1)^{0.5} T/\pi\, t_1 + (1 - 0.5/\mu)/(1 + 0.7T/t_1) \tag{9}$$

Equation (9) is in error by less than 5 percent over the whole range of values of t_1 from zero to infinity and μ from 1 to infinity. It can therefore be used instead of Fig. 6.

From equation (9) one can readily determing q if p_{max} is given, and conversely.

From equation (9) and the relations between p_{max} and p_o in equations (1) and (3) and Fig. 4, one can determine the value of q for any overpressure directly, as soon as the drag coefficient, the value of μ and that of T are known or can be estimated.

It may be useful to have approximate expressions also for the time to reach maximum deflection. These can be derived more or less logically, but only the resulting expressions are given here:

For t_m greater than t_1

$$t_m/t_1 = p_{max}/2q + (0.091\ T/t_1 + 0.25) / (2\mu - 1)^{0.5} \tag{10}$$

and for t_m less than t_1

$$t_m/t_1 = 2 - \left[(2 - 1/\mu) - 2\ (\mu - 1) / (\mu^2 + 100)\right]\ q/p_{max} \tag{11}$$

These expressions are accurate within 10 percent or better in general.

With the values of t_m, one can take account of other than triangular loading curves by selecting the best available approximation in the form of a triangle to the actual loading curve in the region up to t_m.

Finally we require expressions for the equivalent elasto-plastic resistance curves for use in cases such as that shown in Fig. 5(c). Although an exact expression can be derived for the case where the loading is a pure impulse, this is not particularly useful in practical cases. The following expression appears to work fairly well in general, however:

$$q' = \frac{q_e + q_m}{2(1 + t_1/T)} + \frac{3q_e \quad q_m}{4(1 + T/t_1)} \tag{12a}$$

Equation (12a) is applicable to the negative slope case shown in Fig. 5(c); for the positive slope condition the following simpler relation is applicable:

$$\bar{q}' = (q_e + \bar{q}_m)/2 \tag{12b}$$

Properties of Materials

In making a preliminary design some estimate must be made of the values of the ductility factor to be used. In general this is probably fairly low for wood structures, perhaps of the order of 5 to 10. For steel structures, the factor is generally large, of the order of 50 to 100, except in such instances as may occur with welded connections at low temperatures where brittle fractures may be developed because of the speed of loading. For reinforced concrete the factor depends on the amount of reinforcement, and on the relative amounts of tensile and compressive steel, as well as on the way in which the compressive steel is tied together. Crushing of concrete occurs in compression at a fairly low strain, and causes a large reduction in moment carrying capacity. Some data are available in Reference 7 on the behavior of reinforced concrete beams up to the point of failure.

From the results of the test data reported in Reference 7 it appears that the following rough rules may be used for preliminary estimates:

The ductility factor for reinforced concrete beams is given approximately by the relation:

$$\mu = 0.1A_c / (A_s - A'_s) \tag{13}$$

where A_c is the area of the concrete, A_s that of the tensile steel, and A'_s that of the compressive steel, in the cross-section. However, μ should not be taken greater than 30 for reinforced concrete, whatever equation (13) indicates.

For shear walls, the value of μ is likely to be quite small, much less than for framed structures. Values of the order of 5 to 10 seem as large as should be taken for such elements as shear walls and diaphragms.

The influence of the speed of loading on the yield point of mild steel should be considered. What this influence is precisely on steel members in a structure is difficult to evaluate. Factors of the order of 20 to 30 percent higher than the static yield point seem reasonable, however, for members in the framing, and 30 to 50 percent higher for members in the wall or roof covering.

Conclusion

This paper presents an approach which can be used for making preliminary designs of structures to resist blast loadings resulting from atomic bomb detonations. The procedure involves the selection of the required yield resistance of the structure to resist the blast forces. This yield resistance may be as much as twice the peak blast force for a very brittle structure subjected to a very long load pulse, or as little as 1/20 or less of the blast force for a ductile structure subjected to a very short load pulse.

The duration of the pulse and the ductility of the structure must be considered in the solution of the problem. However, once the necessary yield resistance is determined, the rest of the design is essentially like the ordinary routine for static loads.

After the preliminary design is made, better values of the period of the structure and of its ductility factor can be computed. The procedure given herein can be applied again to determine whether the strength of the structure is adequate. Generally no more refined calculation is necessary.

For unusual cases, either with very brittle structures and unusual shapes of load pulses, or for any cases whatsoever, more refined calculations of structural response can be made. However, these do not imply any better result in so far as the design of the structure is concerned.

For those who seek more elaborate analyses, reference is made to item 8 and to papers by the writer in items 2 and 3 of the list of References.

By way of illustration let us consider the design load factors required for a typical framework of a building and for the resistant wall covering of the building for the same resultant pressure.

The period of the wall covering is likely to be very short, of the order of 0.025 sec., whereas the period of the frame is likely to be of the order of 0.25 sec. The duration of the blast force on both of the elements is of the order of 0.25 sec. If the ductility factors for both elements are taken as 20, then from either Fig. 6 or Equation (9) one finds the result that the required static resistance is approximately 0.4 times the peak pressure for the frame and 0.9 for the wall panel. This illustrates the general result that for wall panel elements which generally have a short period, the static resistance required is approximately equal to the peak blast force, whereas for frame members it is considerably less. Therefore, for equal factors of safety these elements must be designed for very much different yield loads. If they were designed for the same static equivalent load, the wall panels would be much weaker than the frame.

References

1. "The Effects of Atomic Weapons," Atomic Energy Commission, published by U. S. Govt. Printing Office, June, 1950.
2. "Proceedings of the Conference on Building in the Atomic Age," Mass. Inst. of Technology, June, 1952.
3. "Proceedings of the Symposium on Earthquake and Blast Effects on Structures," Earthquake Engineering Research Institute and Univ. of California, June, 1952.
4. "Lateral Forces of Earthquake and Wind," by a Joint Committee of the San Francisco Section, ASCE, and the Struct. Eng. Assoc. of Northern California, Proceedings Separate No. 66, ASCE, April, 1951. (Discussion, Proc. Separate No. D-66, May, 1952)
5. "A Simple Method of Evaluating Blast Effects on Buildings," Armour Research Foundation, July, 1952.
6. "The Response of Simple Structures to Dynamic Loads," by Nancy B. Brooks and N. M. Newmark, Univ. of Ill. Structural Research Series No. 51, April, 1953.
7. "An Investigation of the Load-Deformation Characteristics of Reinforced Concrete Beams Up to the Point of Failure," by J. R. Gaston, C. P. Siess, and N. M. Newmark, Univ. of Ill. Structural Research Series No. 40, Dec., 1952
8. "Methods of Analysis for Structures Subjected to Dynamic Loading, " Report to Physical Vulnerability Division, Directorate of Intelligence, U. S. Air Force, by N. M. Newmark, 17 November 1949, revised 18 Dec., 1950.

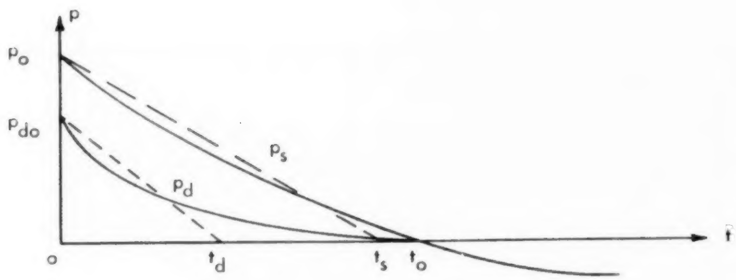

Fig. 1 - Side-on Overpressure and Drag Pressure

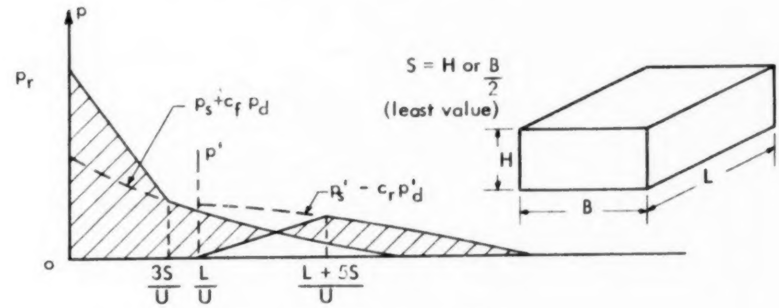

Fig. 2 - Net Pressures on Front and Rear Faces

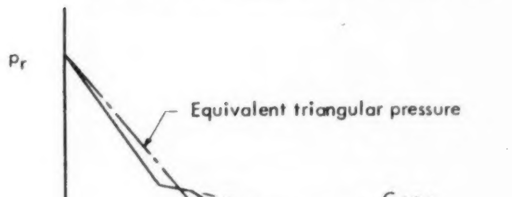

Fig. 3 - Net Translational Force

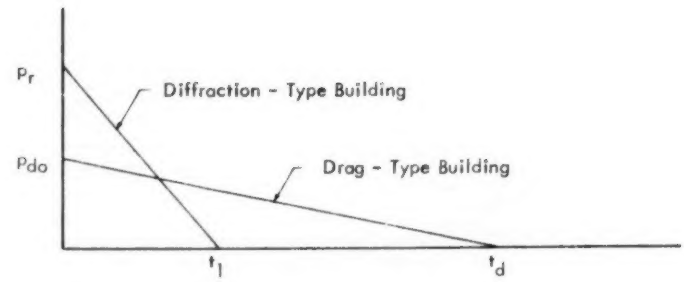

Fig. 4 - Recommended Design Loadings

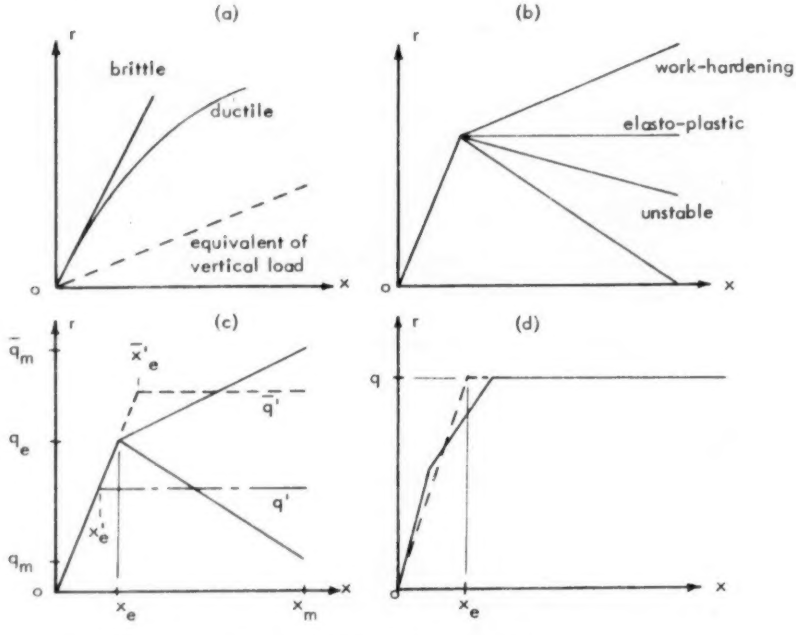

Fig. 5 - Typical Resistance - Deflection Relations

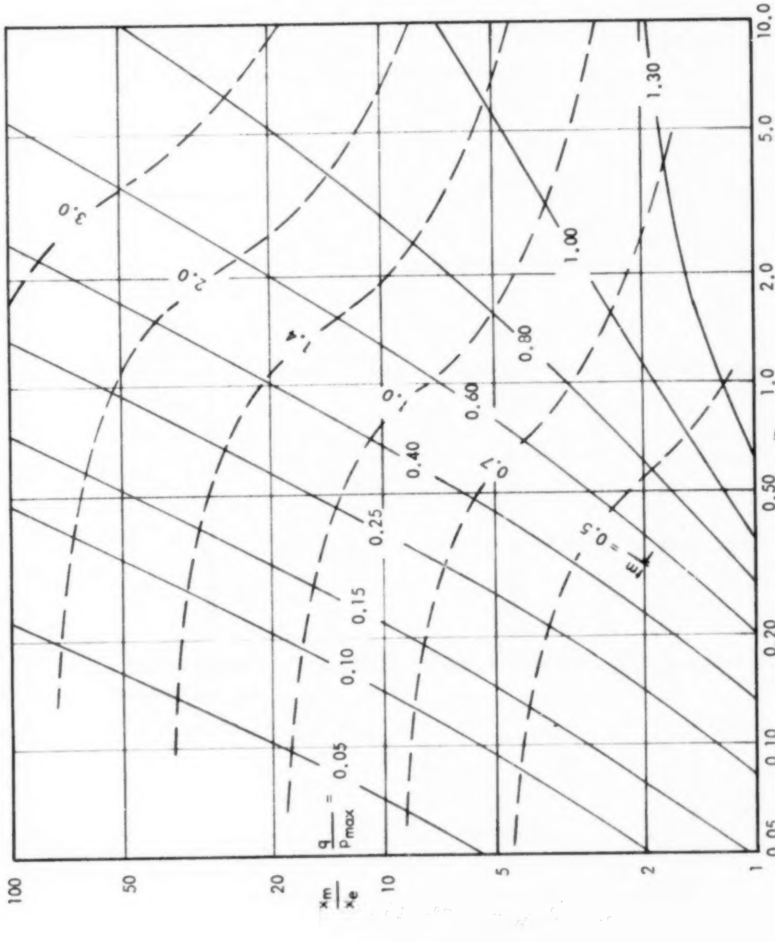

Fig. 6 - Response, Pressure, and Resistance Relations For Triangular Pulse Loading

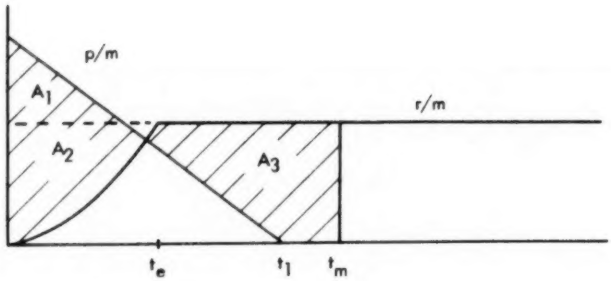

Fig. 7 - Acceleration - Time Relations

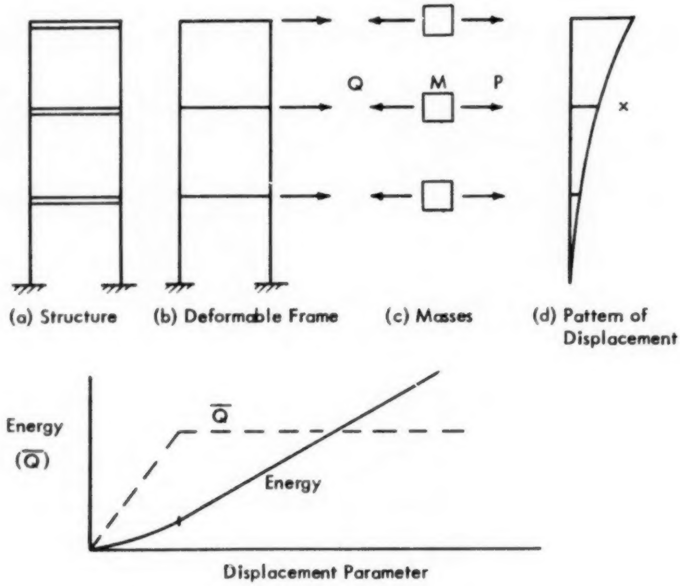

Fig. 8 - Illustration of Analysis of Complex Structure

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1953

PRESIDENT
WALTER LEROY HUBER

VICE-PRESIDENTS

Term expires October, 1953:
GEORGE W. BURPEE
A. M. RAWN

Term expires October, 1954:
EDMUND FRIEDMAN
G. BROOKS EARNEST

DIRECTORS

Term expires October, 1953:
KIRBY SMITH
FRANCIS S. FRIEL
WALLACE L. CHADWICK
NORMAN R. MOORE
BURTON G. DWYRE
LOUIS R. HOWSON

Term expires October, 1954:
WALTER D. BINGER
FRANK A. MARSTON
GEORGE W. McALPIN
JAMES A. HIGGS
I. C. STEELE
WARREN W. PARKS

Term expires October, 1955:
CHARLES B. MOLINEAUX
MERCEL J. SHELTON
A. A. K. BOOTH
CARL G. PAULSEN
LLOYD D. KNAPP
GLENN W. HOLCOMB
FRANCIS M. DAWSON

PAST-PRESIDENTS
Members of the Board

GAIL A. HATHAWAY
CARLTON S. PROCTOR

TREASURER
CHARLES E. TROUT

EXECUTIVE SECRETARY
WILLIAM N. CAREY

ASSISTANT TREASURER
GEORGE W. BURPEE

ASSISTANT SECRETARY
E. L. CHANDLER

PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN
Manager of Technical Publications

DEFOREST A. MATTESON, JR.
Editor of Technical Publications

PAUL A. PARISI
Asst. Editor of Technical Publications

COMMITTEE ON PUBLICATIONS

LOUIS R. HOWSON

FRANCIS S. FRIEL
GLENN W. HOLCOMB
I. C. STEELE
FRANK A. MARSTON
NORMAN R. MOORE